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SEISMIC PERFORMANCE OF BRACED STEEL FRAMES UNDER LONG DURATION GROUND MOTIONS

A. Hammad [(1)], M.A. Moustafa [(2)]

[(1)] *Graduate Research Assistant, Department of Civil and Environmental Engineering, University of Nevada Reno, United States. ali.hammad@nevada.unr.edu*

[(2)] *Assistant Professor, Department of Civil and Environmental Engineering, University of Nevada Reno, United States. mmoustafa@unr.edu*

Abstract

Over the past decade, several long duration subduction earthquakes took place in different locations in Chile and Japan. These events highlighted the importance of comprehensive investigation of the effect of the ground motion duration on structural performance. This paper presents a preliminary computational modeling developed in the OpenSees framework to assess the seismic performance of concentrically braced frames (CBF) under long duration earthquakes. The objective of the study is to explore the effect of low-cycle fatigue on structural members and the accumulated damage as related to the ground motion duration. Nonlinear time history analysis was conducted using subduction and crustal ground motion records. A calibrated model was used to compare between the behavior of a single-story X-braced CBF and a typical six-story frame under both types of ground motions. It was found that the effect of duration on the nonlinear seismic response is particularly significant in terms of the strain accumulated in the braces causing induced low-cycle fatigue fracture. Therefore, particular attention should be given to the effect of earthquake duration when designing CBF buildings located in the proximity of subduction zones and do not rely only on response spectral values in predicting the structural response.

Keywords: Low-cycle fatigue; Braced frames; Long duration ground motions.

1. Introduction

Special concentrically braced frames (SCBFs) are widely used in seismic design. Their strength and stiffness result in an economical system that easily meets serviceability limit states for performance-based seismic design (PBSD). During large infrequent earthquakes, SCBFs must assure life safety and collapse prevention performance states, which requires the simulation of the nonlinear behavior. As a result, analytical models for PBSD used in design must reliably predict both elastic and inelastic performance. However, modeling approaches used in practice might not accurately predict the post-yield mechanisms and failure modes, especially under special lateral loading cases such as long duration earthquakes.

The inelastic seismic response of SCBFs is dominated by compressive buckling, tensile yielding and post-buckling behavior of the braces. The braces are typically connected to beams and columns through gusset plate connections, which must tolerate large inelastic deformations and end rotations associated with brace buckling and sustain the full axial resistance of the brace. Gusset plate connections and beam and column framing members provide boundary conditions to the brace, and therefore influence its resistance and deformation capacity. Accurate SCBF components modeling must consider large deformations for simulation of both local and global buckling. Some studies extended the models to consider the initiation of cracking and fracture based upon the strains, components of strains, or stress–strain history computed in the analyses. This accurate modeling of SCBF might require to develop three dimensional (3D) solid finite element models (FEM), which is substantial computational effort when thousands and even millions of degrees of freedom are involved. Nonlinear analyses of these complex 3D models required considerable computing time with analyses extend for several days only to complete a cyclic load history [1]. Thus, simpler FEM that consider a combination of one-dimensional (1D) force- or displacement-based beam column elements and various translational and rotational springs are computationally appealing. Modeling simplifications might compromise the accuracy of nonlinear analyses used to predict the SCBF seismic performance for instance when compared to detailed 3D FEM. However, a well-calibrated FEM that utilizes only 1D elements can still be very beneficial and provide a lot of insight towards PBSD. This study utilized the latter type of a simplified but relatively accurate nonlinear FEM developed using OpenSees [2] platform. The model used fundamental concepts of engineering mechanics to estimate the properties of its key components and was then calibrated using experimentally measured performance of braced frames. The validity of the overall geometric and component modeling assumptions is verified. However, the suitability of certain material modeling aspect pertaining to the low-cycle fatigue as it relates to earthquake loading cycles and ground motion durations is investigated in this study.

The long duration ground motions that have occurred recently in Iquique, Chile (Mw 8.2, 2014), Tohoku, Japan (Mw 9.0, 2011) and Maule, Chile (Mw8.8, 2010) are a reminder of the importance of the effect of ground motion duration on structural response [3]. The size of the faults rupture controls the durations of the motions in these earthquakes. The 2010 Chile Earthquake ruptured over 500 km and many sites across Chile experienced ground motions lasting for up to 90 seconds. Current seismic design codes do not consider duration effects and rely mainly on spectral accelerations. This might be attributed to the lack of availability of actual recorded long duration ground motion along with the debate of whether synthetic records should confidently replace recorded motions. Recently over the past few years, a large number of long duration motions have been recorded after events such as Tohoku. This led to few studies started investigating the effect of earthquake duration on buildings damage and collapse risk (e.g. [4,5]). One type of buildings that has not been properly studied under long duration earthquake despite its vulnerability is braced steel buildings. SCBF is commonly used in the United States and is particularly susceptible to low-cycle fatigue induced seismic damage. The objective of this study is to investigate the effect of earthquake duration on seismic performance of steel braced frames with special focus on low-cycle fatigue modeling effects. A one-story and six-story SCBF are used in this study to conduct nonlinear time history analysis using two different ground motions that represent one short crustal and another long subduction earthquake events. A brief literature review, description of the developed FEM, and discussion of the analysis results are presented next.

2. Literature Review

The influence of earthquake duration on structural performance does not only depend on the duration definition, but is also highly dependent on the structural model and the engineering demand parameter or damage metric used in judging the performance. Studies using peak response have generally found no correlation between ground motion duration and structural damage [6, 7]. However, studies using cumulative damage measures [7] or energy measures [8, 9] have found a correlation between duration and damage. Marsh and Gianotti [8] used artificial acceleration records representing the Cascadia subduction zone earthquakes as an input for inelastic response history analyses of single degree of freedom systems. They found that structures subjected to long duration motions accumulate damage as a result of repeated cycles.

Raghunandan et al. [4] carried out nonlinear dynamic analyses on 24 buildings designed according to outdated and modern building codes for the cities of Seattle, Washington, and Portland, Oregon. They concluded that that the median collapse capacity of the ductile (post-1970) buildings is approximately 40% less when subjected to ground motions from subduction, as compared to crustal earthquakes. Also they concluded that the seismic performance assessments in which building collapse fragility is quantified based only on crustal motions may substantially underestimate the seismic risk in regions with a subduction hazard. Foschaar et al. [9] investigated the effect of ground motion duration on the collapse capacity of a 3-story steel braced frame. They used two record sets, one with long duration records and the other with spectrally equivalent short duration records. This approach was useful in isolating the effect of the ground motion duration from other ground motion characteristics. They found that the duration affects the collapse capacity significantly. Tirca et al. [10] studied the effect of mega-thrust subduction records versus crustal records by analyzing the nonlinear response of 4-story, 8-story and 12-story Moderately Ductile-Concentrically Braced Frame (MD–CBF) office buildings located in Canada using OpenSees. They found that in the case of the 8-story and 12-story MD–CBF buildings, the largest demand occurred mostly at the same floors; dominant at the bottom two floors under both crustal and subduction record sets. However, in the case of the 4-story MD–CBF building the largest demand occurred at the top floor under the subduction record set and at the bottom two floors under the crustal record set. They concluded that subduction records characterized by longer Trifunac duration did not affect the peak interstory drift or the peak residual interstory drift. Also the damage was concentrated at floors characterized by lower brace over strength. To avoid the occurrence of brace fracture when subjected to ground motions scaled to match the design spectrum, it is recommended to provide a sufficient safety margin when the size of bottom floor braces is selected. Boomer and Martinez-Pereira [11] reported more than 30 definitions of strong ground motion duration in the literature. The most common duration definitions are the bracketed and the significant duration. The bracketed duration is defined as the measure of the time interval between the first and last exceedance of an absolute acceleration threshold, for example 0.05g or 0.1g. The significant duration is defined as the interval over which a specific amount of Arias Intensity (IA) is achieved, this amount is usually taken as 5 to 95% or 5 to 75% of the Arias Intensity.

To complement existing literature, this study focused on brace yielding to fracture in ductile CBF that are designed according to current code and steel design standard as it relates to crustal versus subduction earthquake scenarios such as the mega-thrust Mw9 Tohoku earthquake. Analytical framework is extended to emphasize the effects of Trifunac duration on the nonlinear response of CBF multi-story buildings in terms of low cycle fatigue of braces using detailed numerical models developed in OpenSees as discussed next.

3. Analytical Model

3.1 Model Description

Selected braced frames from previous experimental studies were used in this study such that the developed analytical OpenSees models can be accurately verified and calibrated. Thirty single-story frame specimens were tested at the University of Washington (UW), as illustrated in Fig. 1. These braced steel frames included two beams, two columns, and a single or double brace(s) and were subjected to an axial load and reversed lateral cyclic loading. The laboratory constrained the testing to idealized boundary conditions and did not permit inclusion of floor slabs, but the relative simplicity of the setup permitted evaluation of a wide range of gusset plate configurations, design strategies, brace types and configurations. Two specimens were chosen for this study and were modeled in OpenSees to verify the suggested model, named as HSS1 & HSS31. The specimens are described in Table 1. The models captured tensile yielding, buckling and post-buckling behavior of the brace, which are highly nonlinear behaviors and key elements in the seismic response of the SCBF system. Significant deformation and yielding of the gusset plate connections are also considered along with local yielding of the beams and columns adjacent to the gusset plate. The OpenSees model used mainly simplified discrete component models including beam–column elements and concentrated springs. The modeling technique is similar to models that have been used by Hsiao et al. [1,12]. Force-based nonlinear beam–column elements with four integration points were used to model the braces, beams and columns. Fiber cross sections were employed, which enable the creation of the various steel cross sections with the assumption of plane strain compatibility. Concentrated spring elements were used to model the connections. The Giuffre–Menegotto–Pinto model designated as Steel02 material was the nonlinear constitutive law used for all members. The OpenSees model also was used to evaluate nonlinear models with pinned or rigid joints at the brace–beam–column connections. The following subsections provide additional detail of the modeling approaches and quantification of the important variables used for each.

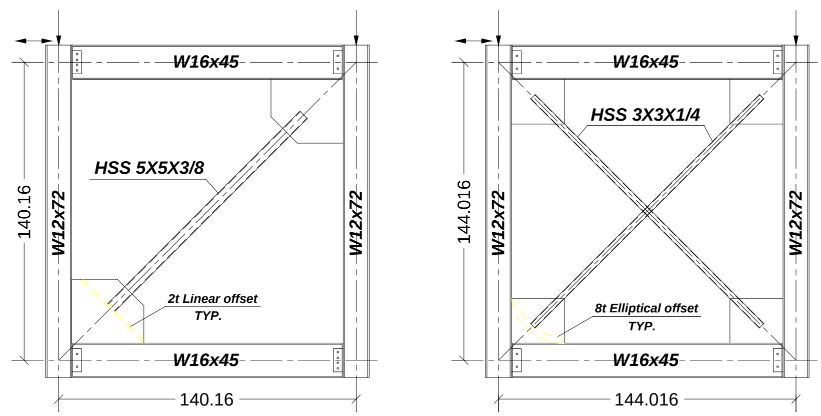


Fig. 1 - Typical test specimen for single-story single-bay SCBF: diagonal-braced HSS1 (left) and X-braced HSS31 (right).

Table 1 – Dimensions and material properties for selected UW test specimens

Specimen	Beam W16*45	Column W12*72	Brace HSS		Gusset Plate				Clearance	
	F_y (ksi)	F_y (ksi)	Size	F_y (ksi)	a (in.)	b (in.)	t_p (in.)	F_y (ksi)	Type	N(t)
HSS1	59.0	59.5	5×5×3/8	69.5	34.0	30.0	0.5	118.0	Linear	2
HSS31	50.0	50.0	3×3×1/4	46.0	11.7	9.25	0.25	87.0	Elliptical	8

3.1.1 Brace

An accurate model of SCBF must include an accurate model of the full cyclic inelastic behavior of the brace, including buckling in compression, yielding in tension and post-buckling behavior. The prior work [13] showed that accurate simulation of brace buckling behavior is achieved with ten or more nonlinear beam–column elements along the length of the brace. Accurate predictions of the American Institute of Steel Construction (AISC, [14]) buckling curves were achieved using an initial displaced shape using a sine function with the apex equal 1/500 of the length of the brace. Measured material properties are provided in Table 1. For the constitutive model, the strain-hardening ratio of 0.01 was used. The brace was stiffer and stronger at the ends as result of the block shear length of the gusset and net section reinforcement. This increased local strength and stiffness was modeled, and reduced deformation with little local yielding occurred in this portion of the brace. To represent the complex cross section at these locations, an increased strength element was used at each end of the brace.

3.1.2 Gusset plate connections

Gusset plate connections in actual structures are neither pinned nor fixed joints, and these connections have a significant effect on the stiffness, resistance and inelastic deformation capacity of the SCBF system. Hence, accurate simulation of these connections is required. To simulate the nonlinear out-of-plane rotational behavior of the gusset plate connections, single and multiple springs along the brace axial direction at and beyond the end of the brace were considered and investigated. Based on what Hsiao et al. [1] have found, the correct estimate of the gusset plate stiffness is required to accurately predict the buckling capacity of the brace. Fig. 2 shows the analytical modeling approach for the gusset plate after Hsiao et al. [1]. The rotational nonlinear spring was located at the physical end of the brace. Rigid links were used to simulate the remainder of the gusset plate, as shown in Fig. 2. The zero-length nonlinear rotational spring element using Steel02 material model at the end of the brace simulated the out-of-plane deformational stiffness of the connection. Therefore, the initial stiffness of this rotational spring was based upon the geometry and properties of the gusset plate. A rational estimate of gusset stiffness is provided based on Equation (1).

$$K^{rot.cal} = \frac{E}{L_{ave.}} + \left(\frac{W_w t^3}{12}\right) \tag{1}$$

where E is Young's modulus of steel, W_w is the Whitmore width defined by a 45° projection angle, L_{ave} is the average of L_1, L_2 and L_3 as shown in Fig. 2, and t is the thickness of the gusset plate. These expressions are empirical with a basis in engineering mechanics.

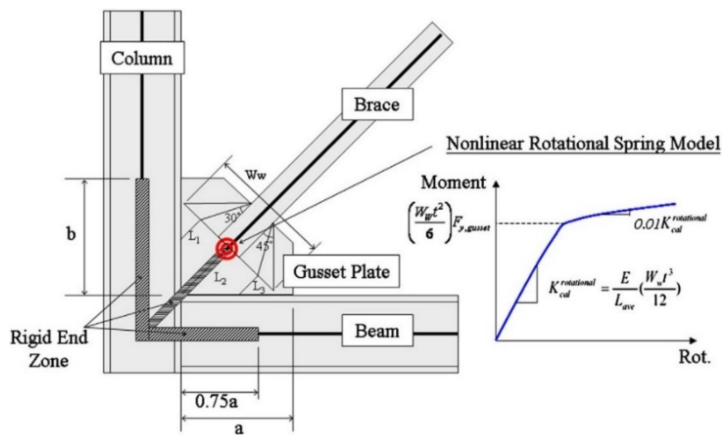


Fig. 2 – Illustration of the proposed connection model after Hsiao et al. [1].

In design, the maximum stress in the gusset plates is approximated using the Whitmore model with a 45° projection angle Fig. 2. Here, the Whitmore width is used to approximate a geometry that provides an equivalent uniform stress distribution, therefore representing the portion of the gusset that is most effective in resisting the loads and deformations of the brace. This width and the gusset plate thickness are used in the computation of the rotational stiffness in Equation (1), which is effectively EI/L. In addition, the flexural strength of the nonlinear rotational spring mode is also based on the Whitmore width. This method is effective in computing the capacity of Whitmore cross section of the gusset plates ($W_w t_p^2/6$). The post-yield stiffness is 1% of the initial rotational stiffness. The gusset plate sustains minimal in-plane deformation relative to other deformations modes of the frame, and therefore rigid links were used to simulate this rigidity.

3.1.3 Beam–column connections

The beam-to-column connections at locations without a gusset plate were shear-plate (or shear-tab) connections. Beam–column connections at beam–column-brace connections were welded flange welded-web connections. As a result, they were treated as fully restrained connections with the simplified pinned or fixed connection models. The connections were simulated as illustrated in Fig. 3a. The model is a combination of the zero-length nonlinear spring element and the Pinching4 material model (both available in OpenSees), which allows to simulate degradation of strength and stiffness as shown in Fig. 3b. The moment-rotation model developed by Liu and Astaneh-Asl [15] was used to estimate the initial rotational stiffness and the maximum positive and negative moment capacities for the shear tab connections with and without slabs. The composite shear-tab connection model includes composite action for shear tabs with composite slabs bearing against the face of the column. All of the parameters of the bending moment and rotation capacity in the figures were based on Liu and Astaneh-Asl's study.

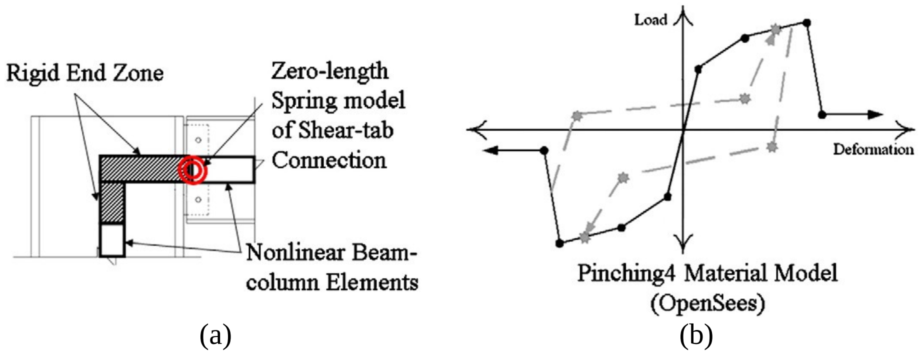


Fig. 3 - Illustration of (a) the shear-tab connection, after [1], and (b) the Pinching4 material model in OpenSees.

3.1.4 Fatigue simulation

To capture the possible fracture of braces due to low-cycle fatigue, the Fatigue material implemented in OpenSees was wrapped around the parent Steel02 material assigned to braces. Once the Fatigue material model is activated, the fibers of HSS brace cross-section start accumulating damage. It is noted that the first brace fracture model implemented in OpenSees was developed by Uriz [16] and is based on low-cycle fatigue of constant plastic strain amplitude, which used an accumulative strain to predict damage in accordance with Miner's rule. However, under seismic loading, the strain may not have constant amplitude [16]. Based on independent research by Manson [17] and Coffin [18], it was concluded that the relationship between the plastic strain amplitude, ε_i, that is experienced by each cycle i, and the number of cycles to failure, N_f, is linear in the log–log domain, while the slope is equal to m. The mathematical expression is given in Equation (2) where ε_0 is the strain for a single reversal. The m and ε_0 parameters are also known as the fatigue ductility exponent and fatigue ductility coefficient, respectively.

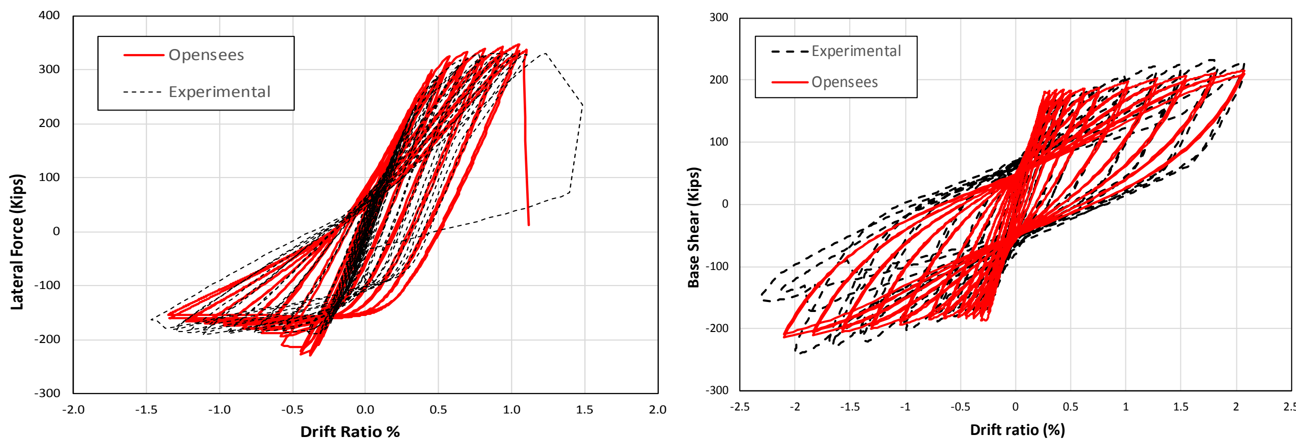
$$\varepsilon_i = \varepsilon_0 \, (N_f)^m \tag{2}$$

Lignos and Karamanci [19] proposed an equation for predicting the value of strain for a single reversal parameter $\varepsilon_{0\ pred}$. The drawback of this equation is that it is applicable for slenderness ratio of a maximum value of 85. To account for HSS braces with kL/r between 50 and 150, Tirca et al. [10] used calibration data from 14 experimental tests to modify the $\varepsilon_{0\ pred}$ equation, as shown in Equation (3), to be used for the fatigue model definition in OpenSees along with m = −0.5.

$$\varepsilon_{0\ pred.} = 0.006\left(\frac{kL}{r}\right)^{0.859} \left(\frac{b_o}{t}\right)^{-0.6}\left(\frac{E}{F_y}\right)^{0.1} \tag{3}$$

3.2 Calibrated model results

Starting from the modeling and input parameters described above, few iterations for calibrating the full model against experimental results were conducted. Several iterations for m & ε_0 values were used to capture the onset of failure. Other important modeling factors that influence the behavior of the brace is the material model input parameters for Steel02 in OpenSees. The developed OpenSees models for diagonal-brace (HSS1) and X-brace (HSS31) frames were analyzed under same cyclic loading history as experiments. The analysis and tests results were compared to investigate the accuracy of the proposed OpenSees model and finalize model calibration. Figs. 4a and 4b show the force-deformation relationships (hysteresis) for calibrated HSS1 and HSS31 models, respectively. Both figures show that the models successfully reproduced the observed behavior from the corresponding experimental test. It is noted that comparable nonlinear displacements and drifts histories were obtained from both the proposed numerical model and the experimental tests but not shown here for brevity. To ensure that the compression response was accurately simulated in the numerical model in both tests, the out of plane buckling value was compared with the experimental tests and showed very close agreement. The proposed model was verified to simulate the behavior of the single story CBF under cyclic loading based on the different model variables calibration against the experimental results. Thus, the model was further used to introduce input ground motions and study the effect of ground motion duration on the behavior of the SCBF with focus on the accumulated damage due to the low-cycle fatigue.



(a) (b)

Fig. 4 - Simulated and measured responses of specimens (a) HSS1 and (b) HSS31.

4. Ground motion selection

To select a representative long subduction and short crustal earthquakes, a set of two spectrally matched ground motions that was used in a recent experimental study at the University of Nevada, Reno [3] was selected. The long duration motion is adopted from the Tohoku 2011earthquake (station FKSH20 N-S) with a significant duration (5-95% of AI) of 88 seconds, and the short duration motion is adopted from the Loma Prieta 1989 earthquake (station Bran 00) with a significant duration (5-95% of AI) of 9.0 seconds. The short duration motion was modified to match the response spectrum of the long duration one. The target was to impose displacement demands on the braced frame model that is equivalent to around half of the maximum displacement capacity of the HSS31 CBF, which was found to be almost 12 inches. The spectral matching of the two motions was done for a period range from 0 to 3 seconds. Thus, the difference between the two motions used in analysis was their durations. The response spectra of the two matched ground motions are shown in Fig. 5. The motions spectra are also compared to the IBC design earthquake (DE) level and maximum considered earthquake (MCE) level for a site in Seattle where the Cascadia subduction zone risk exists.

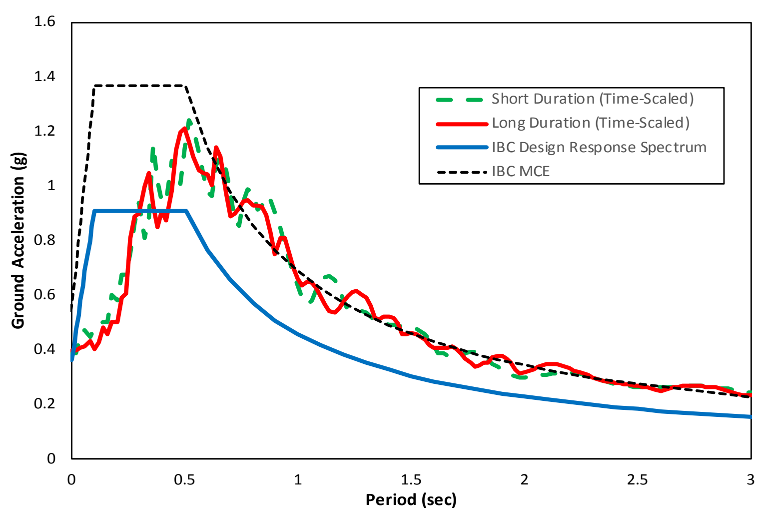


Fig. 5 - Response spectra of the modified long and short motions used in the analysis and corresponding code-based spectra

5. Results and discussions

To study the effect of earthquake duration, the calibrated model for single-story X-braced CBF (HSS31) was used to conduct nonlinear time history analysis using the ground motions previously mentioned. The study was extended also to a typical six-story frame. The analysis results and discussion of the single-story and six-story frames case studies are presented in this section.

5.1 Single story CBF

This subsection summarizes the damage states for the CBF (HSS31) after each applied ground motion. For the long duration ground motion, the CBF reached its final damage state after applying 100% of the main motion. The final damage state refers to the brace experiencing fracture due to low cycle fatigue which, in turns, is attributed to the huge number of cycles accumulated. However, for the short duration ground motion, the 100% of the main motion didn't cause any significant damage at both global and local behavior levels. The damage index (DI) of the braces, which indicates the low-cycle fatigue induced failure, did not show a big value for any damage accumulation cycles. The failure with the short duration motion did not occur until 180% of the main motion was applied and the braces fractured due to fatigue. The fatigue in this case is attributed to a sudden big jump in the displacement and corresponding strain level in the brace, which accumulated a big stress range in one time. Fig. 6 shows a relation between the damage index of the brace versus the drift ratio for both of the 100% scale long and short duration ground motions. The figure shows that damage occurred only in the long duration case because of large number of cycles strain accumulation. The global behavior is compared in Fig. 7, which shows a comparison between the hysteresis of the 100% scale short and long duration ground motions. To capture damage under the short duration case, the input was amplified up to 180% and failure is shown in Fig. 8. Fig. 8a shows the brace DI whereas Fig. 8b compares the global force-displacement relationship from the 100% and 180% scale short ground motion. In Figs. 7 and 8, the final damage state and full collapse is indicated by the loss of the CBF force capacity.

5.2 Six Story CBF

5.2.1 Model Description

To better investigate the effect of earthquake duration on steel frames, multi-story buildings are more practical cases that need to be considered. This study extended the developed and calibrated single-story model to a multi-story building. The configuration and number of stories were preliminarily chosen based on some recent studies that investigated the seismic performance of multi-story steel buildings under particular seismic hazard levels (e.g. 10% probability of exceedance in 50 years) at a specific site. To be specific, a study of modern three- and six-story code compliant concentrically and buckling-restrained braced frames by Sabelli et al. [20] in metropolitan Los Angeles was used here. The prototype structures [15] were designed according to the 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures [21] and the 1997 AISC Seismic Provisions [14]. For the SCBF systems, hollow square sections were employed for the braces with different brace configurations. A six-story frame with X-braced configuration was selected from the study by Sabelli et al. [20] and modeled based on the assumptions and calibrated parameters sued in the single-story case.

The developed six-story model and representation of its first six mode shapes obtained from OpenSees are shown in Fig. 9. The six story building has a typical 13-ft. story-height, but with an 18-ft. height at the first story. Its nominal plan dimensions are 154 ft. × 154 ft. with 30-ft. × 30-ft. bays employed [20]. Floors and roof have a 3-in. metal deck with normal weight concrete topping. The model seismic performance and structural behavior was studied under the selected ground motions previously discussed in Section 4. Viscous damping ratio was assumed to be 5% and modeled using Rayleigh damping.

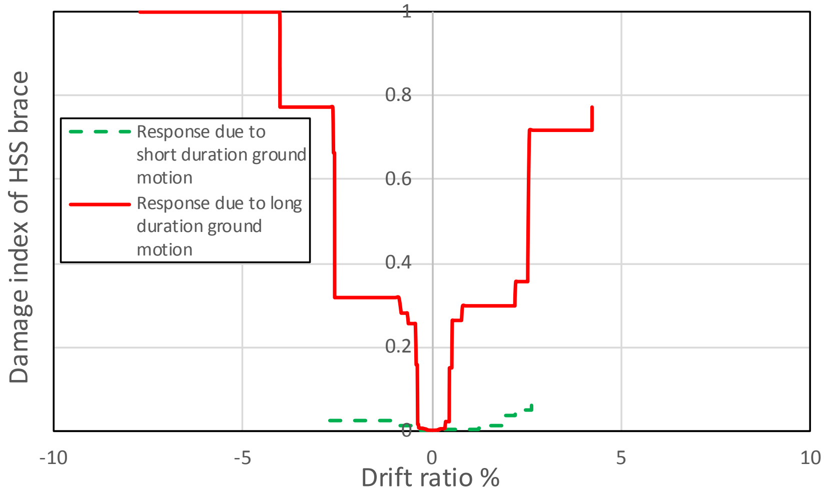


Fig. 6 – Single Story CBF behavior with respect to both short and long ground motions.

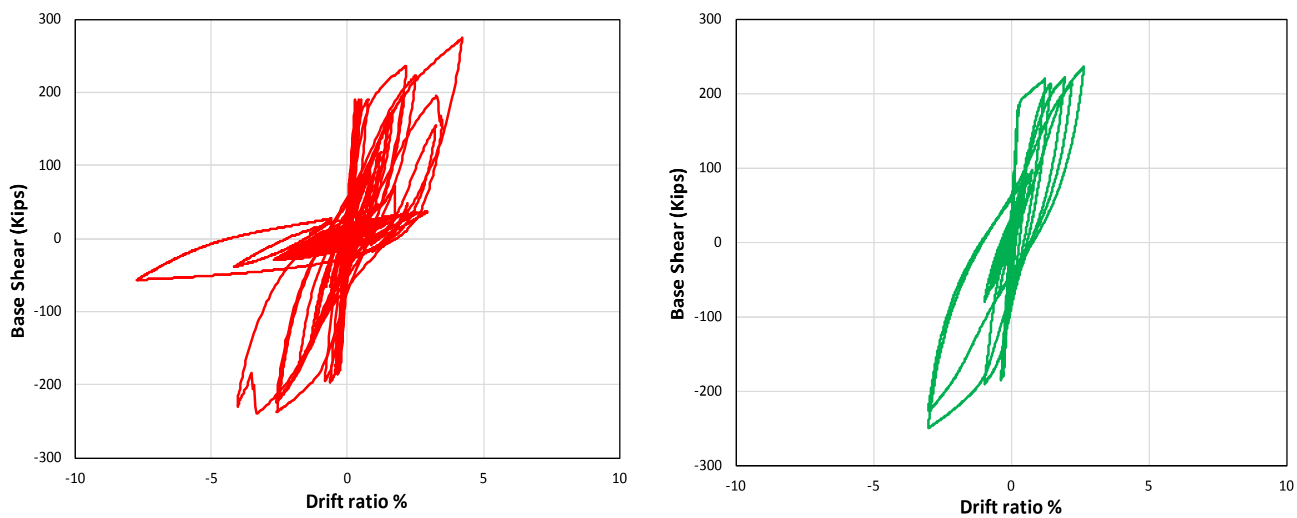


Fig. 7 – Single Story CBF force-deformation relationship (hysteresis) due to the 100% scale of short (right) and long (left) ground motions.

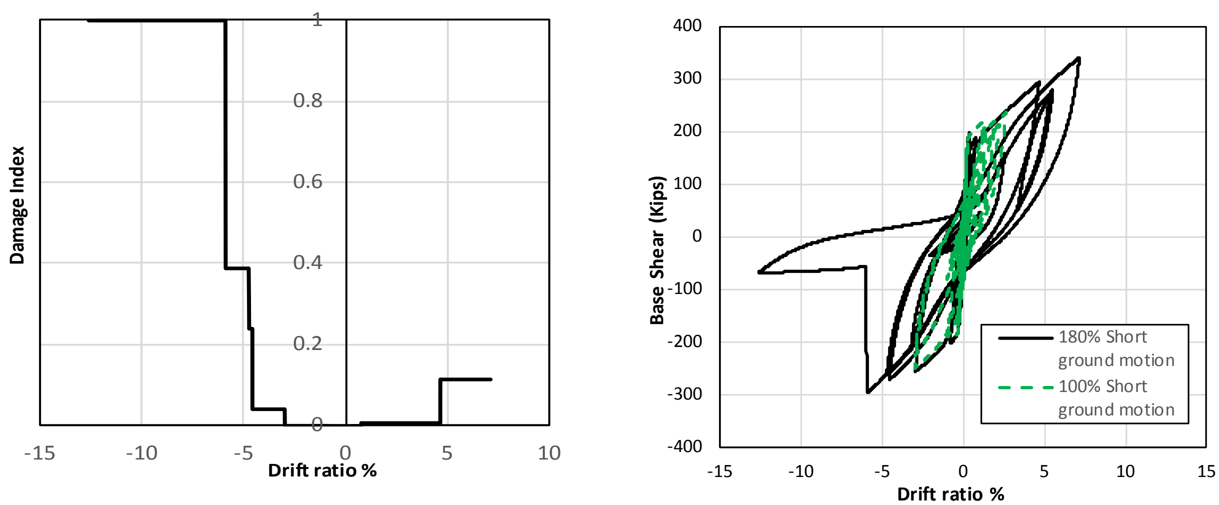


Fig. 8 – Single story CBF response to 180% scale of short duration ground motion.

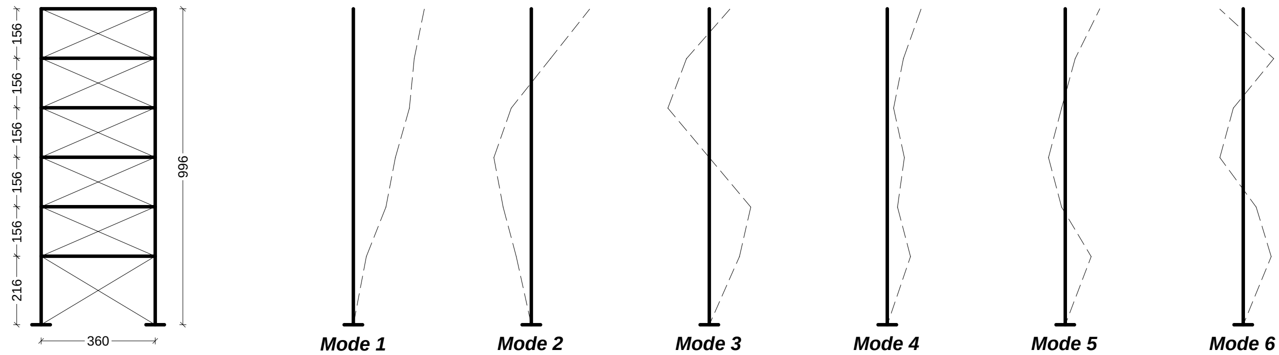


Fig. 9 - Six story (X-braced) CBF under study and the first six mode shapes.

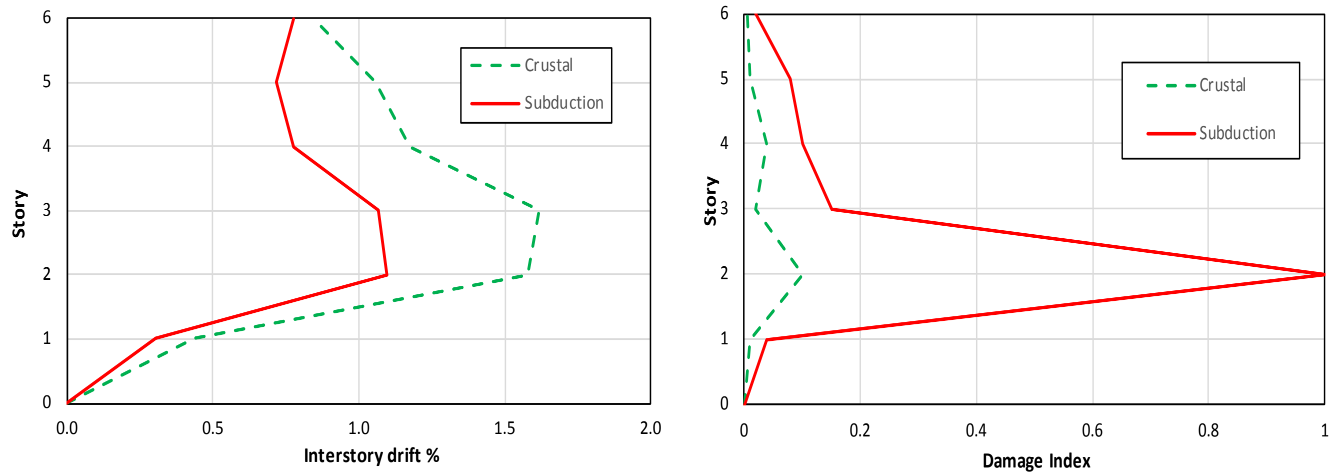
5.2.2 Nonlinear dynamic response

Only selected results from the nonlinear time history analysis of the six-story X-braced CBF building are shown here for brevity. An important demand parameter in design of multi-story buildings is the interstory drift ratio. In this study, the largest interstory drift demand occurred at the 2nd floor under both cases of 100% scale long subduction and short crustal ground motions. The demand was mostly concentrated at the 2nd floor. However, both of the HSS X-braces reached fracture due to low-cycle fatigue only under the long duration subduction record. The interstory drift profile along the building's height obtained under both subduction and crustal ground motions is presented in Fig. 10a. It is clear from the figure that the peak interstory drift occurred at the second floor but higher drift ratios were observed under short duration ground motion even without any brace failure. This preliminarily suggests that the building can fail well before its drift capacities are reached only because of the premature brace failure associated with the long duration earthquake accumulated fatigue damage.

5.2.3 Damage Index of HSS brace

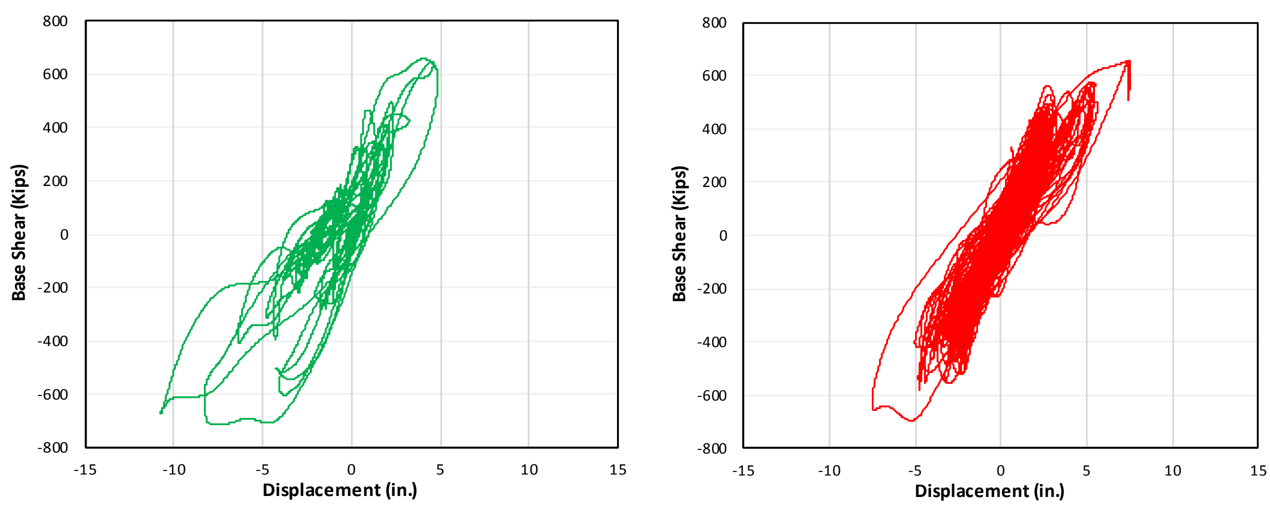
The desirable failure mode of CBFs is the ductile fracture of the HSS braces due to low-cycle fatigue [10]. The strain time-history series recorded in the outermost compression fiber of the HSS brace's cross-section at the level of the plastic hinge, as well as the accumulated strain, are good indicators of brace strain life. In this study, the Fatigue material assigned to each HSS brace was defined using ε_0 resulting from Equation (3) and m=-0.5 as previously discussed. In OpenSees, the number of cycles was counted "on-the-go" with a modified rain flow counting algorithm. The accumulated damage at the level of each HSS brace's cross-section fiber reflects that the damage index was retained to denote the ratio between the current number of cycles and the maximum number of cycles permitted by Coffin–Manson's fatigue theory. Once the damage index of the monitored fiber reached 1.0, the fiber was disconnected. In OpenSees, the fracture of a brace occurs when all fibers of the critical brace's cross-section reach a damage index equal to 1.0. After that, the HSS brace is disconnected from the CBF system.

The largest strain accumulated in CBF brace's cross-section fibers occurs at floors experiencing the peak interstory drift. The damage index was computed for the outermost compression fiber of HSS brace's cross-section at each floor and compared for the two ground motion cases in Fig. 10b. The subduction record causes larger damage at the upper floors relative to the short duration case that excite mostly the lower floors. The damage index of ground floor brace's cross-section is lower than that of the 2nd floor due to the increase of brace cross-section at the bottom floor. The interesting observation is while the magnitude of peak interstory drift is larger, the DI in the different HSS brace's cross-section fibers significantly differs, i.e. a full damage (DI = 1.0) was observed in case of the long duration subduction record (see Fig. 10b). For similar earthquake intensity, the studied six story (X-braces) CBF experienced similar peaks of strain history or strain range under both record sets. However, under subduction ground motion, the accumulated compressive strain in the outermost compression fiber of the HSS brace is about two times larger than that resulting from the crustal ground motion set. Thus, the brace damage index incorporating strain accumulation causing low cycle fatigue is a reliable parameter. Fig. 11 shows the force-displacement (at the roof) relation under both crustal and subduction ground motions.



(a) (b)

Fig. 10 – (a) Interstory drift profile and (b) Damage Index of the outermost compression fiber of HSS braces, under both subduction and crustal records



(a) (b)

Fig. 11 – Six Story CBF force-displacement (hysteresis) due to both (a) crustal and (b) subduction records

6. Summary and Conclusions

SCBFs are one of the most common structural steel systems used as seismic resisting systems. Advanced use of these systems requires accurate yet practical computational models. To enable performance-based seismic design of steel braced frames, a nonlinear analysis modeling approach was developed and verified against experimental tests. The proposed model enabled accurate simulation of the cyclic behavior of the braced frames while minimizing computing cost and time, thus enabling nonlinear dynamic simulation of these important structures. The calibrated models were used in this paper to present preliminary results of investigating the effect of ground motion duration on the structural performance of CBFs, which provide the following conclusions:

- The accumulated plastic strains in the braces subjected to long motions causes the braces to fracture early even if the maximum displacement demands are low. Accordingly, ground motion duration is expected to have a significant effect on the predicted collapse capacity of the buildings using CBF as a structural system.
- The results illustrate that considering only response spectral values in predicting the response of structures can be misleading. In other words, ground motion duration, as a proxy for ground motion type, need to be included in design and assessment of steel structures, especially where long duration motions are expected. This is

because the capacity to absorb energy is significantly reduced with extended ground motion durations causing early fatigue damage.

7. References

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